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INVESTIGATION OF THE DEAD LOAD STRESSES IN THE MISSISSIPPI RIVER VETERANS MEMORIAL BRIDGE AT EAST ST. LOUIS, ILLINOIS

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SYNOPSIS

This paper is a report on measurements of dead load stresses made on certain selected members in the East St. Louis Veterans Memorial Bridge over the Mississippi River. The purpose was to study the actual stress distributions to be expected in the members of large bridges of modern design and construction. The work was performed during the summer of 1949 and the winter and fall of 1950, while the bridge was under construction.

The measured stresses were lower than the computed dead load stress by about 1000 psi. Large stresses and stress concentrations were measured locally in truss splices. It was found that the Engineers and Contractors had been largely successful in eliminating high secondary stresses on the chord members near the channel piers.

Stresses probably due to differential temperature and other erection conditions were found to be in the neighborhood of 3000 psi, and to exist in both horizontal and vertical planes.

INVESTIGATION OF THE DEAD LOAD STRESSES IN THE MISSISSIPPI RIVER VETERANS MEMORIAL BRIDGE AT EAST ST. LOUIS, ILLINOIS

A Joint Investigation by Purdue and Northwestern Universities

In Cooperation with
Committee on Measurement of Dead Load Stresses
American Society of Civil Engineers

Conducted by

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Introduction

The construction of the East St. Louis Veterans' Memorial Bridge offered an unusual opportunity to obtain information on the actual stress distribution in the members of a large modern highway bridge fabricated and erected under the best conditions of modern practice. The dead load stresses are about 75% of the design stress for the large members. The test models are full scale. The testing machine is the structure itself. The results should give some index to the effectiveness of present procedures in design and construction. Because of the importance of information of this type to designing Engineers, and the scarcity of such data members of the staff of Purdue and Northwestern Universities proposed a program for measuring the stresses in this bridge, and comparing the calculated with the measured stresses at several critical sections. The project was financed directly by the two universities and Research Corporation jointly. It was made possible through the generous cooperation of the Bethlehem Steel Co. and of Hazelet and Erdal, executive officers of G. A. Maney & Associates, Inc., Engineers in charge of the bridge design and construction.

Description of Bridge

The new bridge across the Mississippi River connecting St. Louis and East

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(3) Research Associate and Instructor of Structural Engineering, Purdue University.

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St. Louis consists of a cantilever structure with a main center span of 364 feet including a 324 foot suspended span, and two anchor spans of 470 feet each. See Fig. 1. The structure provides a 40 foot roadway to carry H-20 loading. The design and construction of this bridge was conceived and promoted by the late Professor George A. Maney, M. ASCE, with L. T. Wyly, M. ASCE, as principal assistant. The general contractor was Wm. J. Howard, Inc. The cantilever structure was fabricated and erected by Bethlehem Steel Co. The project has been described briefly by Mr. Craig P. Hazelet, M. ASCE[(a)], and by Mr. A. L. R. Sanders, M. ASCE[(b)].

Purpose

The purpose of the investigation was to study the following:

1. The actual dead load stresses at selected sections of the main members entering the lower chord joint above the shoe at the channel pier.
2. Measurements of the closing of milled butt joints and measurements of the stresses in the splice plates for both shop and field splices for some main members under dead loads.
3. The efficacy of the practice of attempting to eliminate the secondary stresses in the main chords by inducing a reversed bending during construction.
4. Any stresses which might be residual in the members as a result of construction or erection conditions.

It was desired to ascertain the stress distribution across the member at sections free from the local stresses delivered by rivets in gussets and splices, and also at sections through the center of the cover plate perforations.

Computations showed that rather large secondary stresses in the bottom chords, approximately 65% of the primary, would be expected to occur in the lower chords adjacent to the river piers if no corrective measures were taken. An attempt to eliminate these was made by inducing reversed bending through the shop and field operations. It was desired to obtain a check on the success of this procedure.

It was also desired to ascertain whether any stresses of appreciable magnitude were induced in these main members by the erection operations.

Fabrication and Erection Procedures

The trusses were cambered to produce geometric outline under full dead load. Under this condition the entire bottom chord is a straight level line. The lengths of all members were corrected in the shop so as to produce the geometric position of the structure in the field under full dead load. Assembly reaming of the chord splices in the shop was performed with the chords in their geometric positions, the web members being shifted accordingly; a practice which has been previously adopted on other large structures. During erection if the members are now connected in the field at the proper length,

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(a) "$10,000,000 Toll Bridge Over Mississippi River Financed by East St. Louis, Ill.", by Craig P. Hazelet, March 1950 Civil Engineering, ASCE.

(b) "East St. Louis Veterans Memorial Bridge", by A. L. R. Sanders, September 1952, Volume 78, Separate No. 150 Proceedings of the ASCE.

reverse bending is set up, particularly in the lower chord truss members, so that theoretically, when the span has been swung and is carrying full dead load the members will have no bending stress.

Erection, starting at the anchor piers and proceeding to the center of the main opening, was accomplished by means of a traveller operating at deck level. In the anchor spans, four falsework bents only, were used. They were located at L_2, L_4, L_6, and L_{10}. During erection, adjustment to line and level was accomplished by the use of wedge jacks located at the base of the steel falsework columns, and a short distance above river level. No falsework was used in the main span.

Unequal temperatures of the lower chords during erection created a problem. The bridge axis is east and west. During the period of closing the anchor span the lower chords of the south truss were exposed to the sun's rays while those of the north truss were shaded by the adjacent stringers and other members. This condition caused the span to warp north out of line in a horizontal plane necessitating corrective measures to bring it back into line at L_{14} at the pier. At this stage the falsework bent at L_{10} was the only one in action. Correction was accomplished by lowering the jack situated at the bottom of the bent under the south falsework column at L_{10}, thus producing rotation in its own plane of the entire falsework bent about the north jack as a pivot. This carried the entire bridge south horizontally and also lowered the south truss.

Another factor permanently affecting the stresses is the erection method used in connecting points M_{13} and M_{15} on the anchor and cantilever spans respectively to the middle of the main post U_{14}-L_{14}. It is necessary that these secondary bracing members be in action as erection proceeds over the pier. However, such connecting members will restrain the natural tendency of these points M_{13} and M_{15} to deflect downward and away from the pier horizontally as erection proceeds. If left as originally connected they would cause a permanent redistribution of stress in all members adjacent to the pier quite different from the state of stress contemplated by the designer.

The logical solution would be to cut both bracing members free from the main post at M_{14} after the structure had been completed and all dead loads including the floor slab were in place, thus allowing the structure to deflect to its normal position, and freeing the main post from members tending to pull it out of line. After the structure had been thus allowed to assume its normal position the bracing members could then be permanently re-connected to the post at M_{14}. Where the main post is too slender to safely carry full dead load without any bracing at midpoint during this adjustment, an alternative solution, consisting of cutting only one member free and leaving the other member permanently connected, is sometimes adopted. This alternative solution was used on this structure, the vertical post being cut free from the cantilever span but left permanently connected to the anchor span.

General Program

The program of the investigation was primarily a field study. However, it was supplemented by a considerable amount of laboratory work to clear up problems encountered as the work progressed as well as to calibrate various

instruments used in the tests. In an effort to measure the stresses which might develop in the process of erection and due to the application of the dead load as the bridge was built, readings were taken at three different stages:

1. No load; members on skids in the shop. See Fig. 13.
2. Dead load, steel only, cantilever stage of construction. See Fig. 2.
3. Full dead load; span swung ready for traffic. See Fig. 1.

The first series was taken while the fabricated members were resting on wooden supports in the shop of the fabricating plant. Because of the massiveness of the members and the method of supporting, with the blocks under the points where the test gage lines were located, any bending which occurred while supported in the shop was found by computation to be negligible. These shop readings were considered as the initial readings. A second set of readings was taken in the field when the bridge was partially completed (see Fig. 2) to determine the strains that had developed and also to substantiate the accuracy of the method of readings that was adopted. Final readings were taken after the bridge was completed with the concrete floor in place, but before the bridge was open to traffic.

Readings were taken on the members of both trusses framing into the lower joints at pier 10. See Fig. 3. Two of the sections were through splices; the others were taken through the perforations in the cover plates.

Description of Strain Measuring Instruments

Strain measurements were made with Whittemore type strain gages of 2, 5, and 10 inch gage lengths. The gages and standard bar are shown in Fig. 4. This type of instrument consists of two tubes, one of steel and one of invar, one telescoped within the other and articulated through plate fulcrums, each carrying a conical point which is inserted in a small hole in the steel member. Movement of these points relative to each other is registered by a linear measuring dial indicator, graduated to one tenthousandths of an inch and mounted on one tube. There are no sliding or rotating parts, and no magnification except for the dial indicator. A floating metal case permits one-hand operation and serves as a shield to insulate the instrument from the heat of the hand. This instrument has the advantage of being equally accurate for reading in any position, upside down, sidewise, horizontally or vertically, the only requirement being that a reasonably consistant pressure be applied by the operator in all positions.

Special Studies

The calibration of the Standard Gage dials used on the instruments is shown in Fig. 5. The average range of the dial in reading a gage line and comparing with the standard bar is about 50 dial divisions, and the maximum range encountered is 100 dial divisions. Hence it is evident that errors due to the dial may be encountered at any time as large as one division.

These gages were calibrated with reference to each other by taking readings on a steel bar under tensile loads, and obtaining a stress-strain curve with each gage. Figure 6 shows these curves. The 5 and 10 inch instruments showed very good agreement and readings for the 2 inch gage were about 4 percent lower.

A check was also made on the effect of temperature on the gages and their

standard bars. Readings were taken in a variable temperature room, thermocouples being mounted on both strain gages and standard bars, and both at approximately the same temperatures. Fig. 7 shows the effect of a wide range of temperature, from 60°F. to -20°F, on the readings. Since in not any case in this investigation did the temperature of the gage or the standard bar change more than one degree between readings on the standard, it was not necessary to make any correction for this relation.

All strain gage readings in the field were referred to standard bars. Two such bars were used for each instrument. One consisted of a section of structural silicon steel from the same lot of material as used in the bridge itself, with small hardened inserts for the gage holes so as to minimize wear. The other bar which was furnished by the manufacturer of the gage consisted of mild steel. Each instrument was cross checked with the standard bars at frequent intervals during the tests. Each standard bar was calibrated with respect to all others.

Fig. 8 shows the results of a study of the method of measuring steel temperatures. This study demonstrated that unless a small pad of cotton is placed on top of the thermocouple used to measure temperature of the steel, the temperature read will be, not that of the steel, but a compromise between the temperature of the steel and that of the air. Accordingly the cotton pad on all thermo-couple installations was adopted.

Location of Gage Lines

The make-up of the members at the sections studied and the location of the gage lines is shown in Fig. 9 and Fig. 10.

Testing Procedure

The most difficult problem in the project is undoubtedly the securing of satisfactory holes. This is particularly difficult in silicon steel which is quite hard in spots. Errors in readings will obviously result: if the surface of the metal is not perfectly smooth and flat, if the surface of the metal and the axis of the hole are not normal to each other, if the holes are not truly circular, if the holes are too shallow, or if the axes of the two holes are not parallel.

Strain readings were obtained by three parties, made up of two men in each party, each man acting alternately as observer and recorder. In addition one or more assistants obtained thermocouple readings.

Each man took two or more complete sets of readings at a given section. These readings, to be acceptable, were required to check to a tolerance of plus or minus one division on the gage dial. All readings were taken "blind" so that it was impossible for the observer to remember a previous reading. All readings were referred to the standard bar, and the standard was read before and after a given short series extending over only a few minutes.

In order to obtain an additional check each 10 inch gage line was divided into two 5 inch gage lines, the strains measured in the latter two serving to check the strain obtained in the other. Thus, in effect there were four independent checks on the strain observation.

To minimize temperature errors all shop readings were taken under cover and only on cloudy days, or at night. All field readings were taken at night. Temperatures of the air, standard bars, and of the steel in the members were taken by thermocouples and a continuous record kept during the strain reading

periods.

A typical field record showing the difference between the temperature of the steel gage line and the standard bar is shown in Fig. 11. Since one degree F difference in temperature will give the same strain as 200 psi, it was decided that corrections for temperature would not be made for less than one degree. The maximum difference in temperature recorded between the standard bar and the steel was 2 degrees F, and this occurred only in a very few cases. This favorable temperature situation was probably due to the fact that readings were taken at night.

Fig. 12 is a general view of the bridge members in the shop, and shows the operations of drilling the gage points. Fig. 13 shows the taking of preliminary initial readings. This photo gives an idea of the size of the members. It will be noted that they are large enough to permit the observer to go inside and take readings on the inside as well as the outside of the sections. Fig. 14 shows a closeup of the lower chord and staging. The regular perforations are 10 inches wide and 20 inches long. The reinforced perforations near panel points are 12 inches wide and 24 inches long. Fig. 15 shows the taking of readings on the bridge in the field.

Probable Errors in Readings

Probably the most common cause of error in readings by mechanical gages is poor gage holes. The silicon steel of which the bridge members were made is quite hard, particularly in certain spots, and it was not always possible to get perfect holes. In some cases holes were redrilled. Even so, a few of the holes were found to be unsatisfactory, and this fact was noted in the records and readings as these holes were discarded.

Considering all probable sources of error, it is believed that the readings reported are not in error more than plus or minus 1,000 psi and that in most cases the error does not exceed half of this figure.

Test Results -- Measured Strains and Stresses

Fig. 16 illustrates how the final stresses were computed from the field record sheets. The readings on the upper half are by one observer and those in the lower half by another.

The measured stresses for both cantilever stage of construction and for final full dead load stage are shown for all truss members studied in Fig. 17 to 26 inclusive. The measured movements of closing of the shop riveted butt joints in chords $L_{13} - L_{14}$ are shown for the cantilever stage of construction in Fig. 27. Comparisons of the measured and computed bending stresses in the vertical planes are given in Fig. 28 and 29 for the cantilever and final stages respectively. Measured bending stresses in the horizontal plane are also given.

The detailed figures for the average measured unit stresses at all gage lines at Section 15 of member $L_{14} - L_{15}$ for the cantilever stage of loading are given in Fig. 17. The stresses are also plotted to scale. In all other cases the stresses are plotted to scale but the detailed readings are not given. E has been taken as 30,000,000 psi.

Since the sections studied are far enough from the truss panel points to be free from the local stress raising effects of the rivets in the gussets and splices, it is to be expected that the stress distribution across the members

would be rather planer. So far as the main sections are concerned, i.e., the four angles and the main web plates, this is found to be true in some cases and not in others. The cover plates obviously tend to act more or less independently of the rest of the member, i.e., the average stress in the cover plate is induced by the stress in the main angles to which it is connected, but the cover tends to act like any other plate with a series of perforations down its center except in so far as it is restrained by the flange angles. Hence it has seemed logical to consider the "average measured unit stress" shown below the figures to be the average of the stress at the four corners of the section and the stress at each corner has been taken as the average of the readings on the edges of the two webs at that corner. The "measured unit bending stress" in the vertical plane has been accordingly considered to be the difference between the above "average measured unit stress" and the average of the measured unit stresses at the top corners of the webs or at the bottom corners of the webs. Similarly the "measured unit bending stress" in the horizontal plane has been taken as the difference between the above "average measured unit stress" and the average of the measured unit stresses at the two corners of the webs on the north side of the member or on the south side of the member.

The following information is shown in Fig. 30 for section 14, member $L_{13} - L_{14}$ south truss for the full dead load stage.

1. Measured unit stresses at all gage lines.
2. Average measured unit stresses at the four corners of the webs and on central planes.
3. Measured unit bending stresses in both vertical and horizontal planes.
4. Computations of the ratios of the stress at the inside of cover plate perforations to the figure obtained by interpolation between the measured stresses in the adjacent web plates.

In Fig. 27 are plotted stresses measured in the webs and also the total movement together of the 2 inch gage line holes on either side of the butt joint measured on the inside of the member. This movement is composed in part of the strain in the main material but mostly it is due to the closing up of the contact bearing surfaces of the milled joint.

Interpretation of Test Results

The average measured unit stresses are generally smaller than the computed stress P/A for the design dead loads by about 1 ksi. In no case are the average measured unit stresses higher than the above design dead load P/A. This indicates that the value of the dead loads assumed by the designers was probably high.

The typical perforations used in these members are 10 inches wide, 20 inches long, and are spaced about 47 to 50 inches c-c. Stresses were measured at these typical perforations at Section 15, in member $L_{14} - L_{15}$ and at Section 14 in member $M_{13} - L_{14}$. The highest measured stress on the inside of such a perforation was at Section 15 of the north truss was 22.2 kips per square inch. This was twice the average measured unit stress for the member at this section. A more representative relation would probably be the ratio between this stress at the inside of the perforation and the value obtained by interpolating between the measured stresses at the adjacent webs.

Such ratios, designated r in this paper, have been computed for all of the measured stresses at the insides of the perforations and are shown in the figures. These ratios so computed vary from about 1.4 to 1.6 for Section 15.

A plate having a single elliptical perforation of the proportions used here, with supported edges and carrying an axial load would be expected to show a somewhat higher stress at the inside of the perforation compared to the axial stress. Two factors act here to reduce this stress concentration:

1. The perforations are spaced fairly closely together, which means that the section of the plate in line with the perforations and located between them will carry a low stress, which in turn reduces the stress at the sides of the perforations. A direct measurement of this action was obtained by the Research staff of the Association of American Railroads in their measurements on the Sante Fe Railroad Bridge at Topock, Arizona. Increasing the distance between perforations would probably result in raising the stress in the cover plate at the inside of the perforations.

2. The cover plate is riveted to the main angles of the chord and these restrain the elastic yielding of the cover plate at the sides. However, it should be noted that the edge distance of the cover plate from the rivet line is 3 inches which puts the rivet line near the neutral axis of the strip of cover plate outside the perforation for bending in a horizontal plane. The consequence of this is that the stress at the outside of the cover plate through the perforations is frequently very different from that in the main webs adjacent. Reducing this edge distance would probably result in reducing the stress in the cover plate at the inside of the hole.

It is to be noted that the highest stress occurs in the plate at Section 15, Fig. 18 which is taken at a typical perforation. It should also be noted that the reinforcing bars added to the cover plate at the sides of the perforations are apparently under riveted, since most of them show a lower stress than the plate they are intended to support.

The measurements indicate the stress to be nearly planer at Section 13 and 15 of the north truss and at Sections 14 of $L_{13} - L_{14}$ and $L_{14} - L_{15}$ and at Section 15 of the south truss. At the other sections the members appear to have undergone some warping. It should be noted that stresses were measured at the middle of the width of the vertical webs, i.e., at points w_o, w_1 on the outside and at w_n and w_s at the inside of the webs for both Sections 14 of the lower chords. See Fig. 9. These measurements show a stress distribution in each web nearly planar across the width of the plate, i.e., across the vertical face.

If the stress distribution across the member had been measured at sections adjacent to the gussets or splice plates, it is highly probable that it would have been found to show high local stress concentration from the rivet loads delivered to the member by the gusset or splice plates.

In Fig. 27 are plotted measurements taken at the shop riveted splice at Section 14S in the lower chord. This splice was designed to carry 50 percent of the stress in the chord, the other 50 percent to be taken in bearing at the milled joint surfaces. In the vertical planes splice plates were provided both

3.Will be published in AREA Proceedings, Vol. 55, 1954.

inside and outside of each main web or rib section. In the horizontal planes splice plates were placed on the outside surfaces only. It was thus possible to take readings on 2 inch gage lines located at 90 degrees to the plane of the milled joint and centered on it, on the horizontal legs of the main angles and on the inside faces of the top and bottom cover plates. See Fig. 10. The readings on these gage lines inside the member at this splice were then a measure of the movement of the contact surfaces of the material closer towards each other, plus any shortening of the material due to stress change. Readings on the 2 inch gage lines located at 90 degrees to the plane of the milled joint and centered on it on the outside splice plates, in both horizontal and vertical planes, were a measure of the shortening of these plates due to stress.

In the opinion of the authors the measurements plotted in the horizontal inside planes in Fig. 27 represent mostly movement of the contact surfaces closer to each other, while the measurements plotted on all outside planes represent strains and stresses in the outside splice plates. It will be noted that both of these readings have been plotted to the same equivalent scale.

The following observations are noteworthy:

1. The measurements show a closing up of the joints to a magnitude of about .01 inch at the north top corner of the south truss chord and somewhat less at the same corner of the north truss chord. See Fig. 27.

2. For both truss splices little or no stress is measured in the horizontal splice plates placed outside the butt joint. For the north truss splice a reasonable stress is measured in the north outside splice plates, but for the south truss splice the measured unit stress in the north outside plates runs to the neighborhood of the elastic limit of the material: also the stress variation across the width of the splice plates in the vertical plane is large. In both trusses little or no stress is measured in the outside south plates.

3. Measurements taken at these shop splices at the full dead load stage show no appreciable change in readings over the cantilever stage readings. This data has not been shown in this paper. It would seem to indicate that the joint has been pretty much closed up on all four sides.

The explanation of the circumstances just mentioned is quite possibly the following:

1. The south web plates and bottom cover plates of each truss chord were brought into contact bearing during shop riveting, particularly at the top, but the north web plates and the top cover plates at the north sides were not brought into full contact bearing at this time.

2. During the loading for the cantilever stage the butt joint closed to full bearing and thereafter little stress would be added to splice plates.

3. The absence of stress in the top and bottom outside splice plates is perhaps due to rivet slip. The grip of these rivets is short.

4. The presence of high stress in the north side plates could be due to the fact that the rivets came to bear after initial slip, as the joint closed up and held. These rivets have a fairly long grip.

5. The greater stress in the splice plates of the south truss may easily be due to the fact that the amount of movement measured in closing the joint is greater than in the north truss.

The implications of these measurements on the riveted shop butt joint designed to carry a considerable part of the load in direct bearing are:

1. If the material is not in complete and full bearing at the time of riveting (a condition not easily secured) slip of the rivets and serious readjustment of the stress distribution among the elements of the splice is likely to occur at an early stage of loading.

2. The material in such a connection is likely to be stressed to above the elastic limit at any time, depending entirely upon the fit of metal in the original connection and the load later placed on the joint.

Complete sets of readings were taken on the gage lines of the field splice at Section 14S of member $M_{14} - L_{14}$ at all stages of construction. See Fig. 10. Here also all readings were checked by a second two-man party, and close agreement was found between the results by all observers at a given stage. In this case also, readings were taken on the outside of the splice plates. All measured stresses were in tension and some were in the elastic limit region. The explanation appears to be as follows:

1. Initial readings in the shop were taken when the splice plates were riveted to the L_{14} joint only.

2. A large part of the dead load stress was placed on the joint in the field, bringing contact surfaces to firm bearing, before the field riveting was done. Therefore only a very small amount of compressive deformation would be caused in the splice plates by additional load in the vertical $U_{14} - L_{14}$.

3. All splice plates at the joint may have been bent outwards, perhaps by the shop riveting, perhaps by accident, prior to taking of initial readings. The pulling of these plates into tight contact with the vertical during field riveting could then produce a large tensile stress on the outside faces of the plates.

4. It seems probable that the heavy clamping of hot driven rivets may set up very large stresses in the material adjacent to the rivets.

The implication of these measurements is again that detail material is likely to be stressed very highly locally.

Several construction operations may be expected to affect the final dead load stresses in the structure. During erection the structure was held closely to proper level and alignment. As has been noted above it was the intention to induce reverse bending in the lower chord truss members adjacent to the channel pier to neutralize the secondary stresses due to dead load. To accomplish this the proper web members had been shortened in length so that the chords would be pulled up in erection.

Two other construction operations, both discussed earlier in this paper, may be expected to affect the final dead load stresses in some of the members. The first is the cutting loose of the bracing member $M_{14} - M_{15}$ from the post $U_{14} - L_{14}$ at M_{14}. The second is the manner of closing the anchor span at the channel pier when unequal temperatures obtained in the lower chords of the north and south trusses.

The degree of success attained in all of these operations may be clearly seen in the stresses found to be residual in the structure at its completion.

In Fig. 28 are plotted the following:

1. Computed bending stresses in a vertical plane due to dead load secondary stresses at the cantilever stage, including the effect of cutting $M_{14} - M_{15}$ loose from M_{15}.
2. Measured unit bending stresses in the vertical plane at the sections under study.
3. Measured unit bending stresses in the horizontal plane for the lower chord members.

In Fig. 29 are plotted the same stresses for the final full dead load stage of construction. The effect upon the secondary stresses of such large truss joints as that at L_{14} of this bridge is considerable. This effect was taken into account by a rational and expeditious method of analysis recently presented and checked against measured stresses for a large bridge by two of the authors.[4]

The following comments are pertinent:

1. Computed secondary stresses at the edge of L_{14} gusset are about 9.3 ksi or 72 percent of the computed dead load P/A stress on the river side and about 6.6 ksi or 55 percent of the computed dead load P/A stress on the shore side. These include the effect of the brace $M_{13} - M_{14}$ in pulling $U_{14} - L_{14}$ out of line towards the shore.
2. The attempt to avoid the high secondary stresses in the lower chords was completely successful for $L_{13} - L_{14}$ in the north truss and for $L_{14} - L_{15}$ in the south truss. In $L_{13} - L_{14}$ for the south truss at Section 14 the bending stress was reduced to about 2.7 ksi or 23 percent of the P/A stress. In $L_{14} - L_{15}$ the bending stress was reduced to a negligible value at Section 14 and reached a value of 3.4 ksi or 26 percent of P/A at Section 15.
3. In two cases the residual bending stresses are opposite in sign to the computed secondary stresses, suggesting that correction was carried a little beyond the intended value.
4. The marked out of plane variation of stress in members $M_{13} - L_{14}$ at Sections 14 could be brought about during riveting if all pins and bolts were removed from one side of the member while it was carrying substantial stress.
5. The influence of the rotation counter clockwise of joint L_{14} due to the pull of $M_{13} - M_{14}$ on the vertical is clearly seen in the measured stress pattern for all members entering this joint. If this brace could be cut free and then reconnected a still better stress condition would result.
6. The lower chord bending stresses in the horizontal plane of 3.1 ksi for

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4 "Rigid Frame Analysis for Members of Variable Moment of Inertia", Parts 1, 2, by L. T. Wyly and K. H. Lenzen. AREA Bulletin 502, June-July 1952, pp. 1-64 incl.

$L_{13} - L_{14}$ at Section 14 on the north truss and of slightly less stress in the other members may be due to difficulties with unequal temperature conditions. They may also be due in part to slippage in the plates in the splices on either side of the truss joint L_{14}.

Conclusions

It seems to the authors that the following conclusions are warranted by the results of their investigations:

1. When properly conducted, dead load stress measurements made under field conditions are entirely reliable and trustworthy. Errors as large as 1000 psi should be expected however.
2. It will be desirable to study further the action of perforated cover plate columns, particularly for ultimate loads.
3. It should be recognized that in joints carrying load partly by riveted splice plates and partly by milled joints in bearing differential local slippage is likely to occur and the splice plates are likely to be stressed into and perhaps above the yield stress range.
4. It is entirely practicable and possible to reduce the dead load secondary bending in the plane of the trusses by changing the lengths of the members, but unless unusual precautions are taken and high grade supervision provided there is no assurance that success will attend the effort.
5. Bending in the horizontal plane due perhaps to conditions of temperature during erection may be expected to exist to the extent of about 25 percent of the axial dead load stress.
6. Where fabrication and erection processes do not attain the standards used on this job the bending stresses in both planes will probably be considerably larger.

Acknowledgement

The authors are indebted to many people for assistance in conducting this investigation:

To the Bethlehem Steel Company and the Bethlehem Officials for permitting the tests to be made, for allowing access to their shops to take initial readings, for furnishing field office space, very substantial stagings and walkways at the job.

To Hazelet and Erdal, executive officers of G. A. Maney and Associates, who assisted in arranging the project, and who helped in its execution by furnishing the cooperation of Resident Engineer N. J. Law and his crew.

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Appreciation is also expressed for the helpful cooperation of Leon D. Urbain, assistant to the Resident Engineer, in providing special facilities throughout the field investigation.

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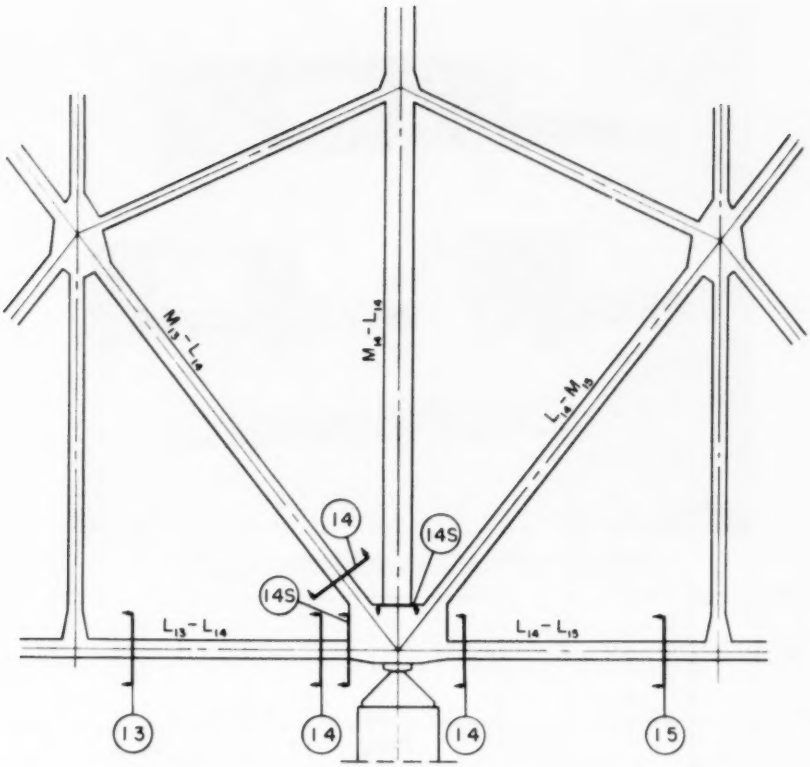
28	Comparison of Measured and Computed Bending Stresses in Truss Members at L_{14} for the Cantilever Stage of Construction.
29	Comparison of Measured and Computed Bending Stresses in Truss Members at L_{14} for Final Full Dead Load Stage, etc., of Construction.
30	Computation of Average Measured Unit Stress, Measured Unit Bending Stresses, and Cover Plate Stress Ration for Section 14-Member L_{13}-L_{14} South Truss, Full Dead Load Stage.

Fig. 1 -- Aerial View of Bridge

Fig. 2 -- Structure at Cantilever Stage



PIER 10

SECTIONS INVESTIGATED

Fig. 3

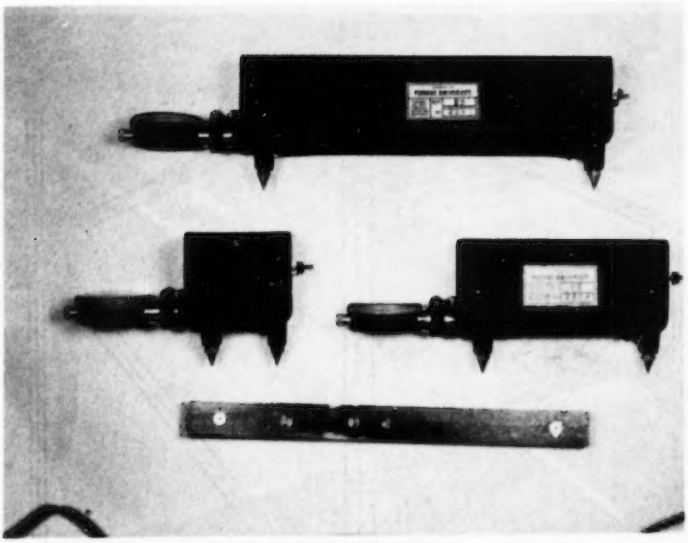
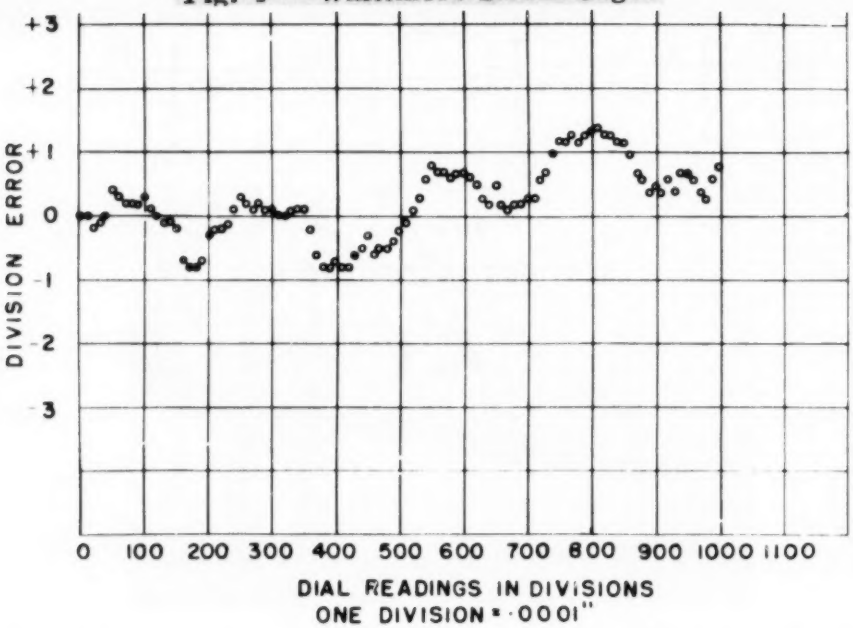


Fig. 4 -- Whittmore Strain Gages



PRATT AND WHITNEY STANDARD MEASURING MACHINE USED
APRIL 22, 1951

Fig. 5 -- Calibration of Dial Indicator

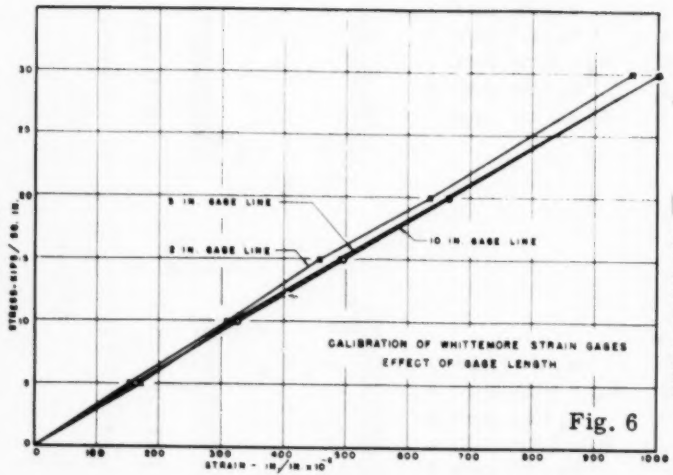

5 IN GAGE LINE
2 IN GAGE LINE
10 IN GAGE LINE
CALIBRATION OF WHITTEMORE STRAIN GAGES
EFFECT OF GAGE LENGTH
STRESS - PSI/IN. IN. 30,000,000
STRAIN - IN/IN x 10⁻⁶

Fig. 6

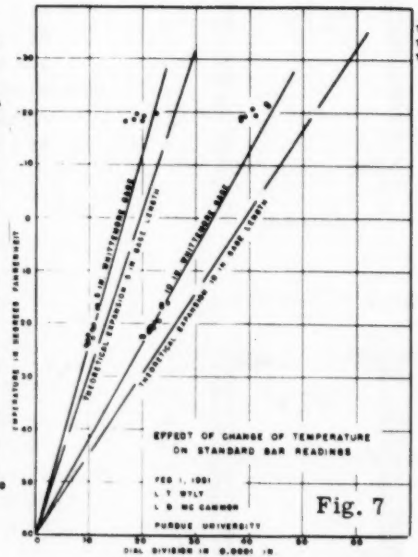


Fig. 7

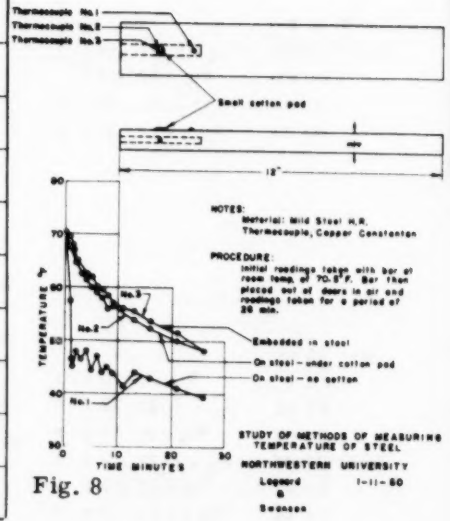
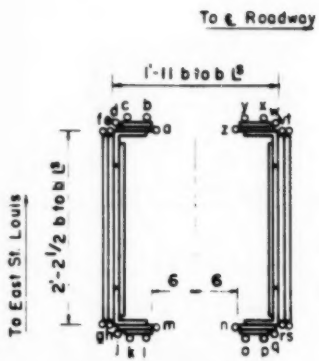
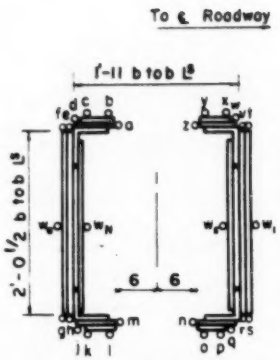


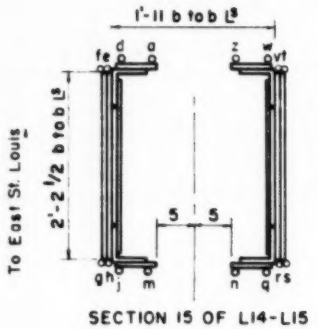
Fig. 8



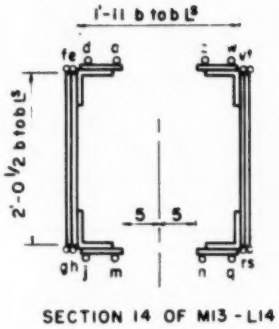
a	4 L's	5 x 5 x 5/8	23.44
b	4 PL's	26 x 3/4	78.00
c	2 PL's	22 1/2 x 5/8	28.12-15.00
d	2 PL's	16 x 5/8	20.00
e	2 PL's	23 1/2 x 9/16	26.44
f	4 PL's	4 x 3/8	6.00
			167.00 sq in



a	4 L's	5 x 5 x 5/8	23.44
b	4 PL's	26 x 3/4	78.00
c	2 PL's	22 1/2 x 5/8	28.12-15.00
d	2 PL	16 x 5/8	20.00
e	2 PL	23 1/2 x 1/2	23.50
f	4 PL	4 x 3/8	6.00
			164.06 sq in

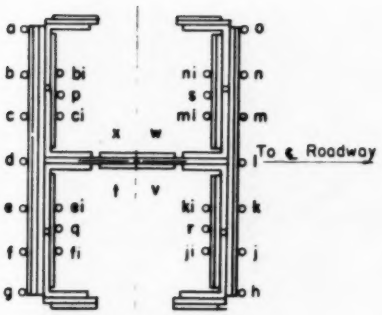


a	4 L's	5 x 5 x 5/8	23.44
b	4 PL's	26 x 3/4	78.00
c	2 PL's	22 1/2 x 5/8	28.12-12.50
d	2 PL's	16 x 5/8	20.00
e	2 PL's	23 1/2 x 1/2	23.50
			160.56 sq in

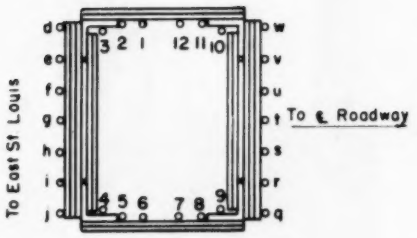


a	4 L's	5 x 5 x 3/4	27.76
b	2 PL's	24 x 3/4	36.00
	2 PL's	24 x 13/16	39.00
c	2 PL's	22 1/2 x 5/8	28.12-12.50
			118.38 sq in

Fig. 9 -- Gage Line Location on Members Tested



SECTION 14S OF L14-M14



SECTION 14S OF L13-L14

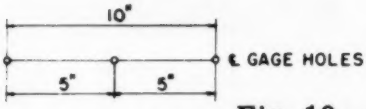


Fig. 10 -- Gage Line Location at Splices

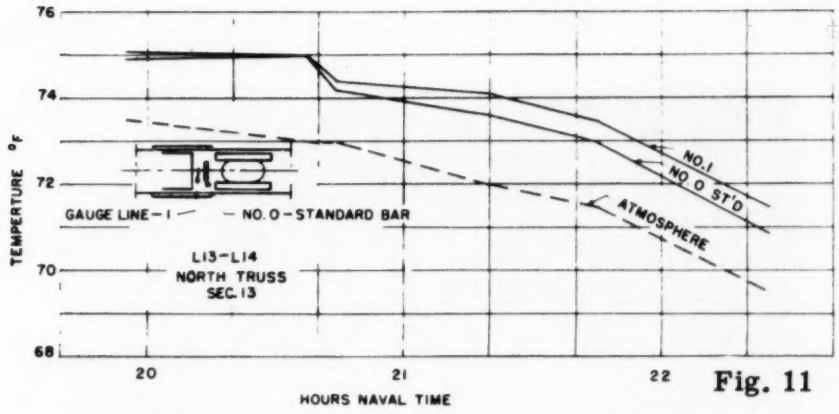


Fig. 11

FIELD THERMOCOUPLE TEMPERTURE READINGS

Figure 12
Drilling Gage Holes in Members in Shop

Figure 13
Taking Initial Shop Readings

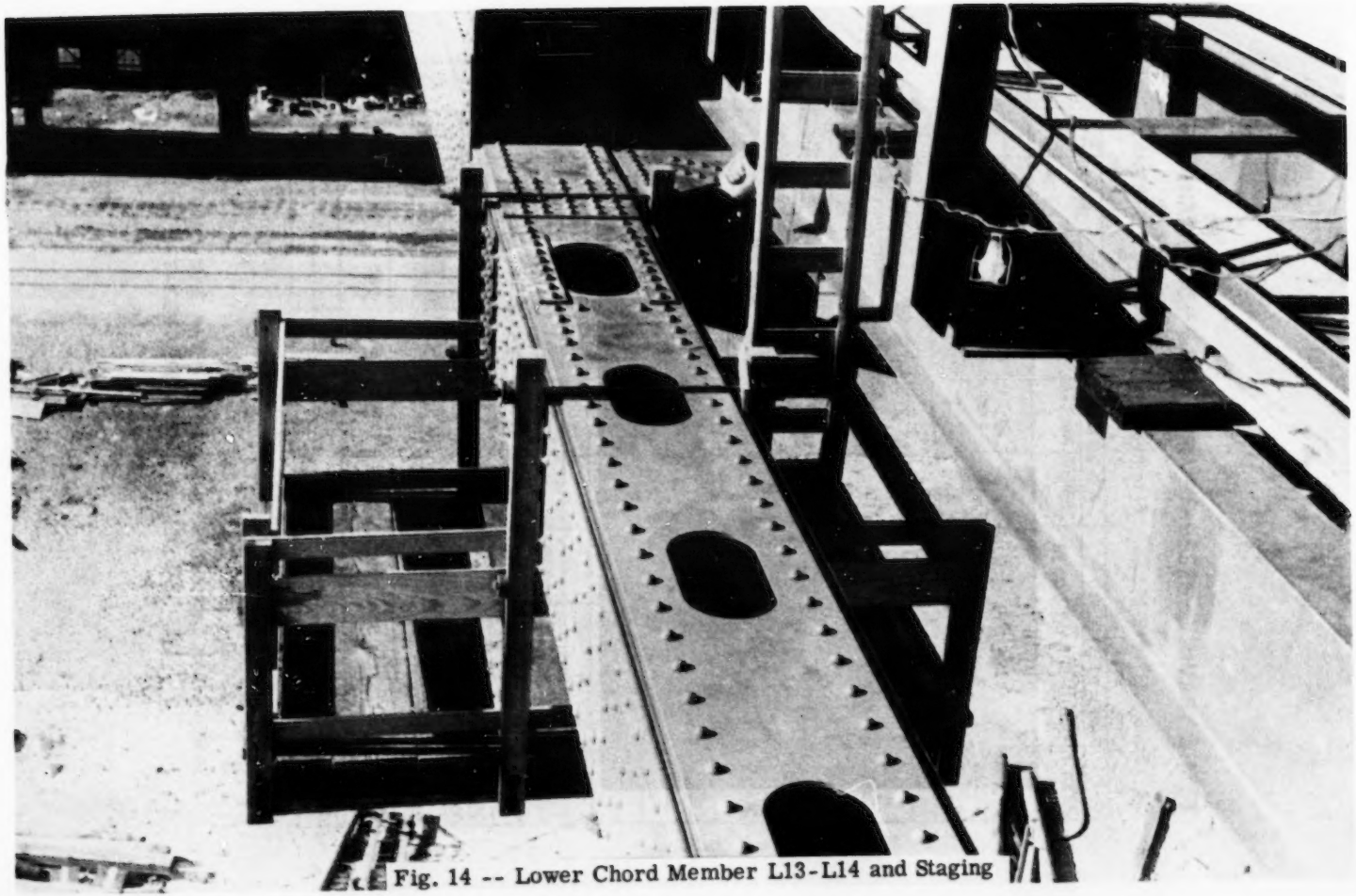


Fig. 14 -- Lower Chord Member L13-L14 and Staging

Figure 15

Taking Readings on Bridge in Field

TERMINATION OF DEAD LOAD STRESSES-SUMMARY SHEET

SPAN CANTILEVER MEMBER L14 - L15 COMPUTER BHATT S J
TRUSS NORTH SECTION L15 CHECKER WYLY

GAGE LINE	GAGE LENGTH	INITIAL READ	STEEL LOADS-CANTILEVER READ	DIFF	STRAIN ϵ	STRESS S	FULL DEAD LOAD READ	DIFF	STRAIN ϵ	STRESS S
LENZEN	2									
	2									
	2									
	2									
	2									
	Σ2									
	5	+589	+385	-204	.000408	-12 240	+305	- 284	.000568	-17 040
	5	- 708	- 893	- 185	.000370	-11 100	- 990	- 282	.000564	-16 920
	Σ5									
	10	- 151	-569	-418	.000418	-12 540	- 700	- 549	.000549	-16 470
	AV.									
WYLY	2									
	2									
	2									
	2									
	2									
	Σ2									
	5	- 701	-903	-202	.000404	-12 120	- 989	- 288	.000576	-17 280
	5	+587	+377	-210	.000420	-12 600	- 312	- 275	.000550	-16 500
	Σ5									
	10	- 152	-580	-428	.000428	-12 840	- 691	- 539	.000539	-16 170
	AV.									

DATE 5-19-51 AV. -12 240 AV. -16 730

Fig. 16

NOTE:
ALL READINGS ARE TAKEN FROM FIELD RECORD AND ARE AVERAGE DIFFERENCES BETWEEN REFERENCE BAR AND GAGE LINE

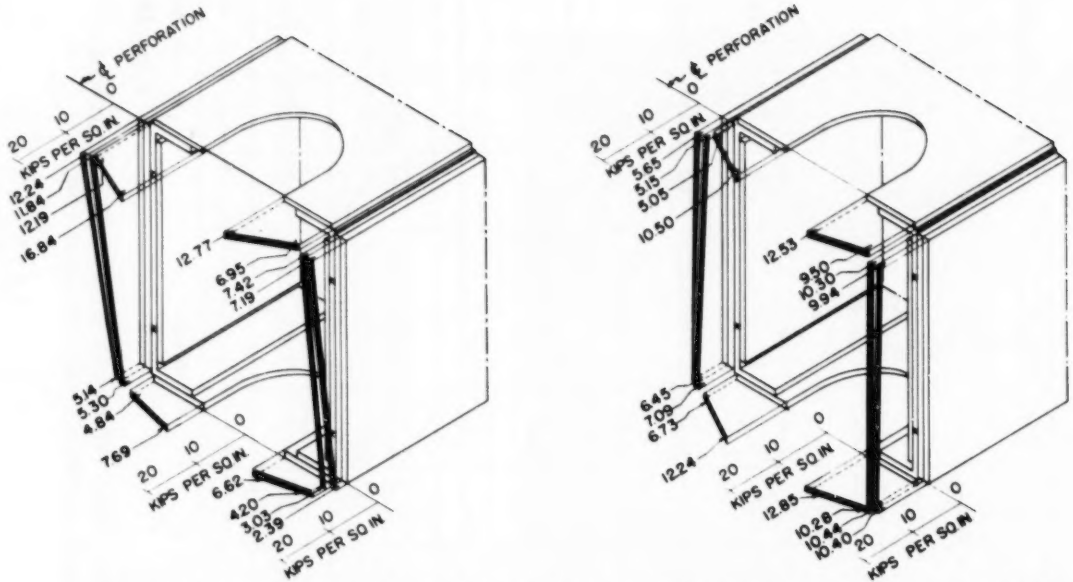


Fig. 17 -- Cantilever Load Stresses at Section 15 in Members L14-L15.

North Truss

DATA: SECTION ON ℄ PERFORATION

GROSS AREA	160.56	SQ IN.
COMPUTED STRESS P/A	9.05	KIPS PER SQ IN.
AVERAGE MEASURED UNIT STRESS	6.81	KIPS PER SQ IN.
MEASURED UNIT BENDING STRESS		
IN VERTICAL PLANE	2.86	KIPS PER SQ IN.
IN HORIZONTAL PLANE	1.81	KIPS PER SQ IN.

South Truss

DATA SECTION ON ℄ PERFORATION

GROSS AREA	160.56	SQ IN.
COMPUTED STRESS P/A	9.05	KIPS PER SQ IN.
AVERAGE MEASURED UNIT STRESS	8.18	KIPS PER SQ IN.
MEASURED UNIT BENDING STRESS		
IN VERTICAL PLANE	0.42	KIPS PER SQ IN.
IN HORIZONTAL PLANE	2.10	KIPS PER SQ IN.

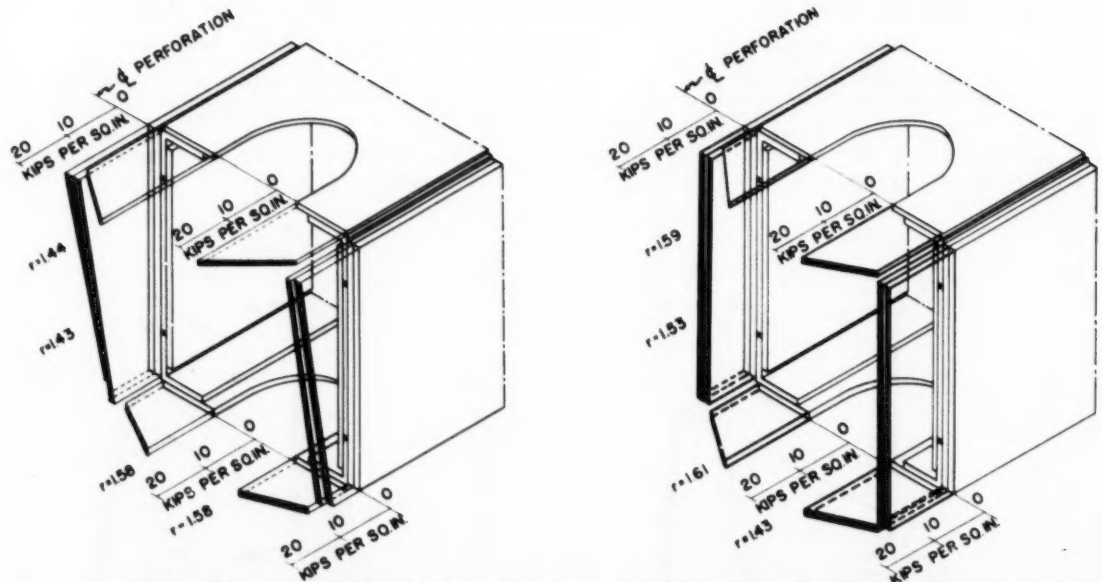


Fig. 18 -- Full Dead Load Stresses at Section 15 in Members L14-L15.

North Truss

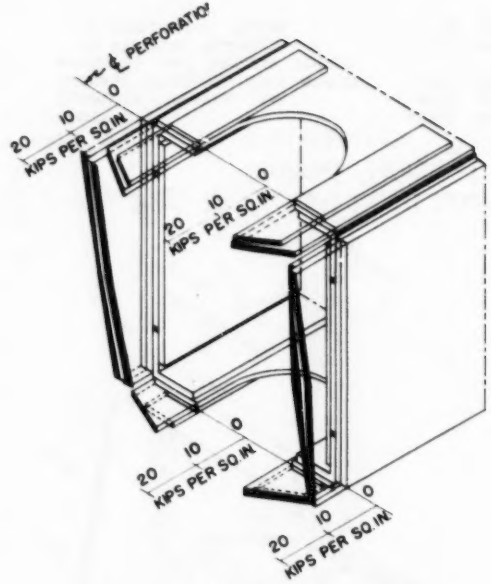
DATA: SECTION ON ℄ PERFORATION

GROSS AREA	160.56	SQ.IN.
COMPUTED STRESS P/A	12.83	KIPS PER SQ.IN.
AVERAGE MEASURED UNIT STRESS	11.06	KIPS PER SQ.IN.
MEASURED UNIT BENDING STRESS		
IN VERTICAL PLANE	3.44	KIPS PER SQ.IN.
IN HORIZONTAL PLANE	1.78	KIPS PER SQ.IN.

South Truss

DATA: SECTION ON ℄ PERFORATION

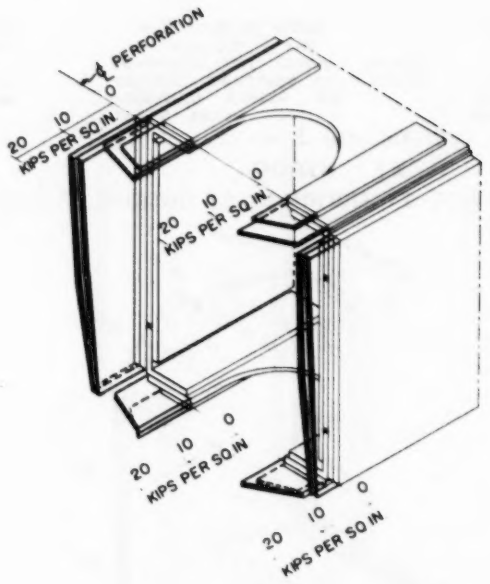
GROSS AREA	160.56	SQ.IN.
COMPUTED STRESS P/A	12.83	KIPS PER SQ.IN.
AVERAGE MEASURED UNIT STRESS	11.38	KIPS PER SQ.IN.
MEASURED UNIT BENDING STRESS		
IN VERTICAL PLANE	0.04	KIPS PER SQ.IN.
IN HORIZONTAL PLANE	2.06	KIPS PER SQ.IN.



North Truss

DATA: SECTION ON ℄ PERFORATION

GROSS AREA	164.06	SQ IN
COMPUTED STRESS P/A	8.84	KIPS PER SQ IN
AVERAGE MEASURED UNIT STRESS	8.27	KIPS PER SQ IN
MEASURED UNIT BENDING STRESS		
IN VERTICAL PLANE	3.00	KIPS PER SQ IN
IN HORIZONTAL PLANE	0.21	KIPS PER SQ IN



South Truss

DATA SECTION ON ℄ PERFORATION

GROSS AREA	164.06	SQ IN
COMPUTED STRESS P/A	8.84	KIPS PER SQ IN
AVERAGE MEASURED UNIT STRESS	8.49	KIPS PER SQ IN
MEASURED UNIT BENDING STRESS		
IN VERTICAL PLANE	1.60	KIPS PER SQ IN
IN HORIZONTAL PLANE	2.70	KIPS PER SQ IN

Fig. 19 -- Cantilever Load Stresses at Section 14 in Members L14-L15.

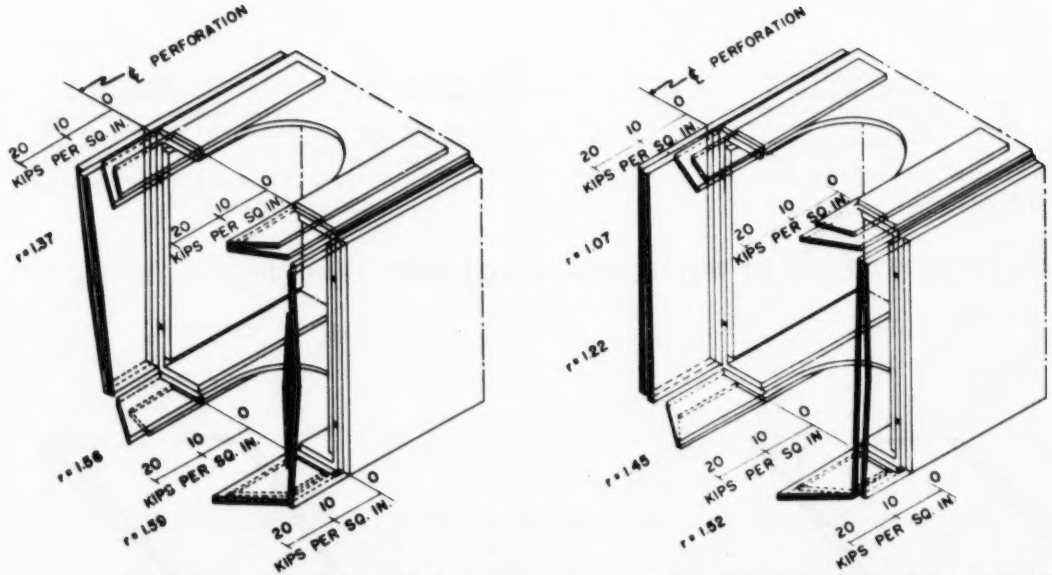


Fig. 20 -- Full Dead Load Stresses at Section 14 in Members L14-L15.

North Truss

DATA: SECTION ON ℄ PERFORATION

GROSS AREA	164.06	SQ. IN
COMPUTED STRESS P/A	12.55	KIPS PER SQ. IN.
AVERAGE MEASURED UNIT STRESS	11.78	KIPS PER SQ. IN.
MEASURED UNIT BENDING STRESS		
IN VERTICAL PLANE	1.12	KIPS PER SQ. IN.
IN HORIZONTAL PLANE	0.58	KIPS PER SQ. IN.

South Truss

DATA: SECTION ON ℄ PERFORATION

GROSS AREA	164.06	SQ. IN.
COMPUTED STRESS P/A	12.55	KIPS PER SQ. IN.
AVERAGE MEASURED UNIT STRESS	12.20	KIPS PER SQ. IN.
MEASURED UNIT BENDING STRESS		
IN VERTICAL PLANE	0.10	KIPS PER SQ. IN.
IN HORIZONTAL PLANE	2.61	KIPS PER SQ. IN.

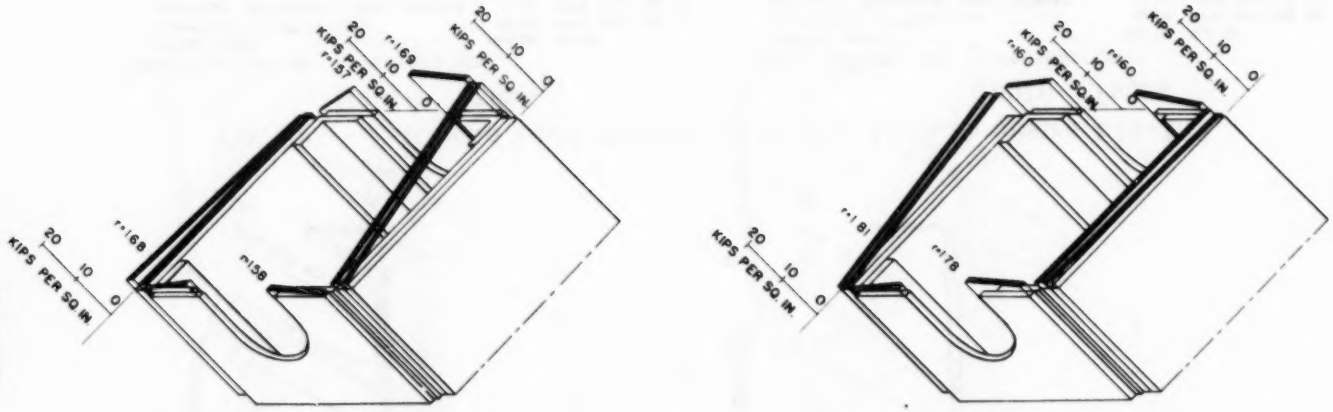


Fig. 21 -- Cantilever Load Stresses at Section 14 in Members M13-L14.

North Truss

DATA SECTION ON ℄ PERFORATION

GROSS AREA	118.38	SQ. IN.
COMPUTED STRESS P/A	3.94	KIPS PER SQ. IN.
AVERAGE MEASURED UNIT STRESS	3.74	KIPS PER SQ. IN.
MEASURED UNIT BENDING STRESS		
IN VERTICAL PLANE	1.92	KIPS PER SQ. IN.
IN HORIZONTAL PLANE	2.06	KIPS PER SQ. IN.

South Truss

DATA SECTION ON ℄ PERFORATION

GROSS AREA	118.38	SQ. IN.
COMPUTED STRESS P/A	3.94	KIPS PER SQ. IN.
AVERAGE MEASURED UNIT STRESS	2.73	KIPS PER SQ. IN.
MEASURED UNIT BENDING STRESS		
IN VERTICAL PLANE	1.39	KIPS PER SQ. IN.
IN HORIZONTAL PLANE	1.17	KIPS PER SQ. IN.

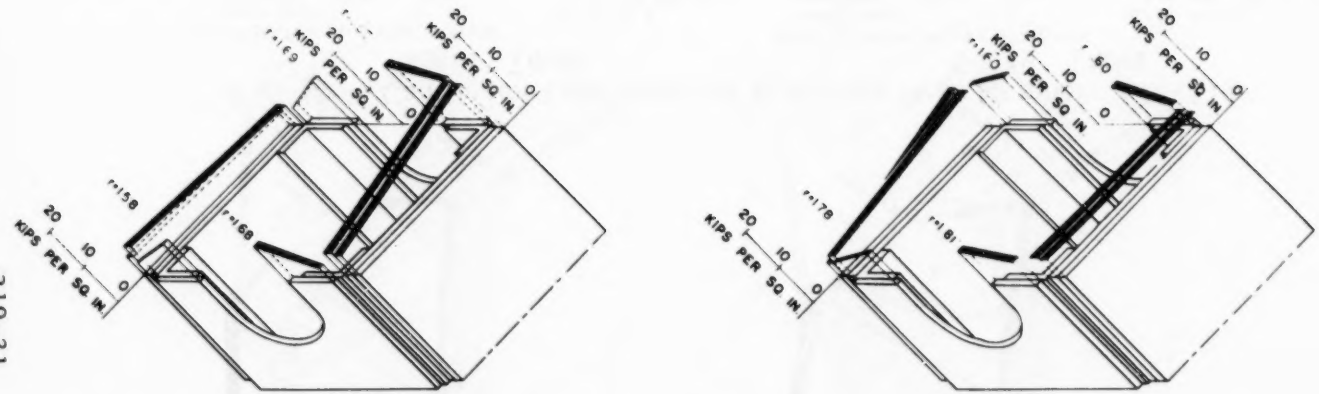


Fig. 22 -- Full Dead Load Stresses at Section 14 in Members M13-L14.

North Truss

DATA SECTION ON ℄ PERFORATION

GROSS AREA	118.38	SQ IN
COMPUTED STRESS P/A	7.75	KIPS PER SQ IN
AVERAGE MEASURED UNIT STRESS	7.37	KIPS PER SQ IN
MEASURED UNIT BENDING STRESS		
IN VERTICAL PLANE	1.25	KIPS PER SQ IN.
IN HORIZONTAL PLANE	1.80	KIPS PER SQ IN.

South Truss

DATA SECTION ON ℄ PERFORATION

GROSS AREA	118.38	SQ IN
COMPUTED STRESS P/A	7.75	KIPS PER SQ IN
AVERAGE MEASURED UNIT STRESS	6.25	KIPS PER SQ IN
MEASURED UNIT BENDING STRESS		
IN VERTICAL PLANE	1.16	KIPS PER SQ IN
IN HORIZONTAL PLANE	1.32	KIPS PER SQ IN

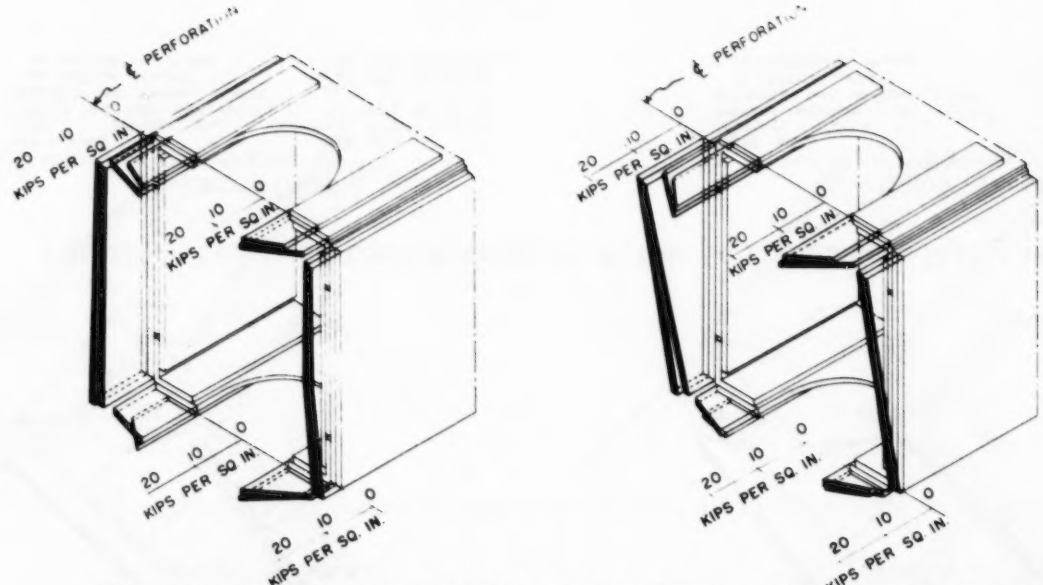


Fig. 23 -- Cantilever Load Stresses at Section 14 in Members L13-L14.

NORTH TRUSS

DATA: SECTION ON ℄ PERFORATION

GROSS AREA	167.00	SQ IN
COMPUTED STRESS P/A	8.74	KIPS PER SQ. IN.
AVERAGE MEASURED UNIT STRESS	7.83	KIPS PER SQ. IN.
MEASURED UNIT BENDING STRESS		
IN VERTICAL PLANE	0.22	KIPS PER SQ. IN.
IN HORIZONTAL PLANE	2.82	KIPS PER SQ. IN.

SOUTH TRUSS

DATA: SECTION ON ℄ PERFORATION

GROSS AREA	167.00	SQ. IN.
COMPUTED STRESS P/A	8.74	KIPS PER SQ IN
AVERAGE MEASURED UNIT STRESS	7.70	KIPS PER SQ IN
MEASURED UNIT BENDING STRESS		
IN VERTICAL PLANE	4.23	KIPS PER SQ IN.
IN HORIZONTAL PLANE	3.41	KIPS PER SQ IN.

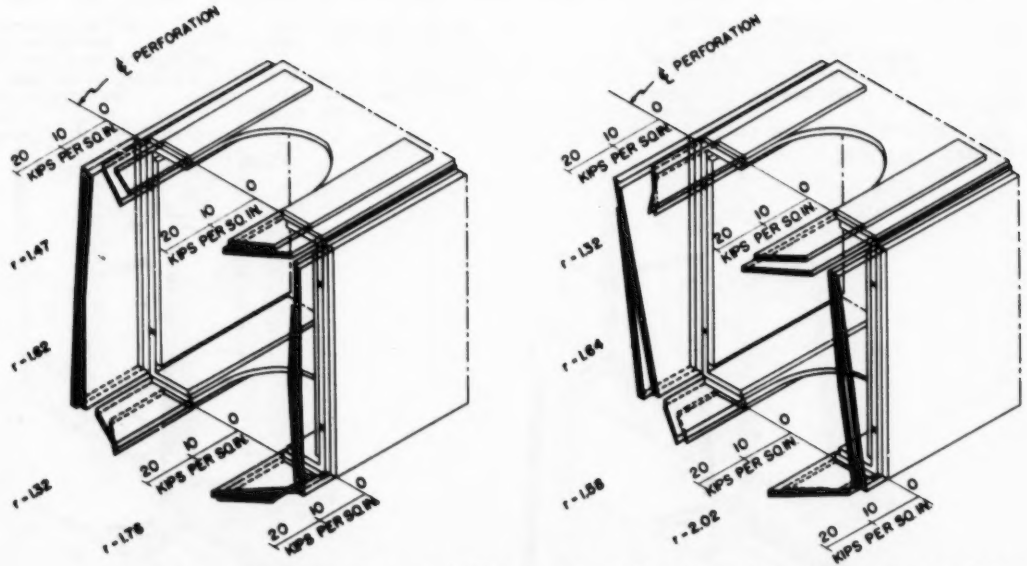


Fig. 24 -- Full Dead Load Stresses at Section 14 in Members L13-L14.

North Truss

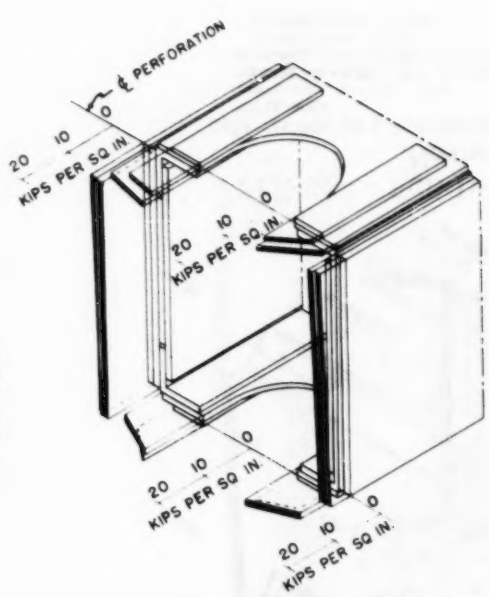
DATA: SECTION ON ℄ PERFORATION

GROSS AREA	167.00	SQ. IN.
COMPUTED STRESS P/A	11.82	KIPS PER SQ. IN.
AVERAGE MEASURED UNIT STRESS	10.03	KIPS PER SQ. IN.
MEASURED UNIT BENDING STRESS		
IN VERTICAL PLANE	0.68	KIPS PER SQ. IN.
IN HORIZONTAL PLANE	3.07	KIPS PER SQ. IN.

South Truss

DATA: SECTION ON ℄ PERFORATION

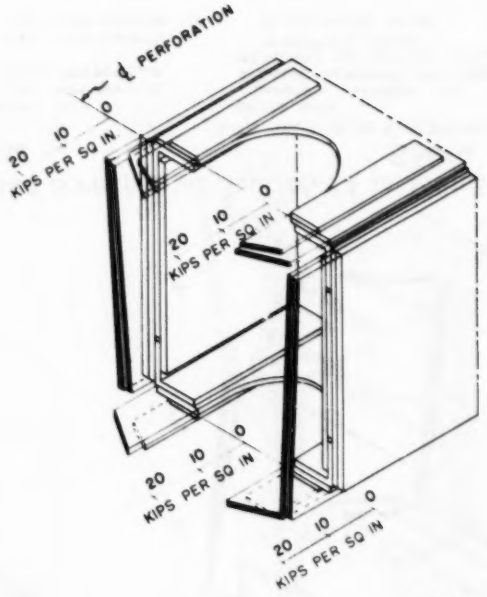
GROSS AREA	167.00	SQ. IN.
COMPUTED STRESS P/A	11.80	KIPS PER SQ. IN.
AVERAGE MEASURED UNIT STRESS	10.83	KIPS PER SQ. IN.
MEASURED UNIT BENDING STRESS		
IN VERTICAL PLANE	2.71	KIPS PER SQ. IN.
IN HORIZONTAL PLANE	2.80	KIPS PER SQ. IN.



North Truss

DATA SECTION ON ℄ PERFORATION

GROSS AREA	167.00	SQ. IN
COMPUTED STRESS P/A	8.74	KIPS PER SQ. IN.
AVERAGE MEASURED UNIT STRESS	8.14	KIPS PER SQ. IN
MEASURED UNIT BENDING STRESS		
IN VERTICAL PLANE	0.50	KIPS PER SQ. IN.
IN HORIZONTAL PLANE	3.12	KIPS PER SQ. IN.

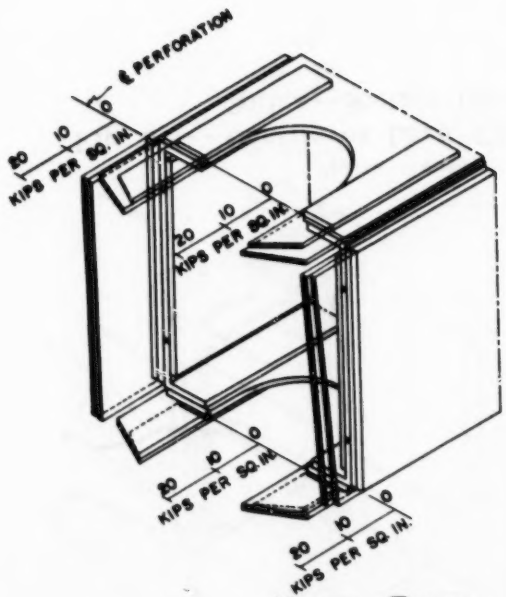


South Truss

DATA SECTION ON ℄ PERFORATION

GROSS AREA	167.00	SQ. IN.
COMPUTED STRESS P/A	6.74	KIPS PER SQ. IN.
AVERAGE MEASURED UNIT STRESS	8.19	KIPS PER SQ. IN
MEASURED UNIT BENDING STRESS		
IN VERTICAL PLANE	0.25	KIPS PER SQ IN
IN HORIZONTAL PLANE	1.96	KIPS PER SQ IN

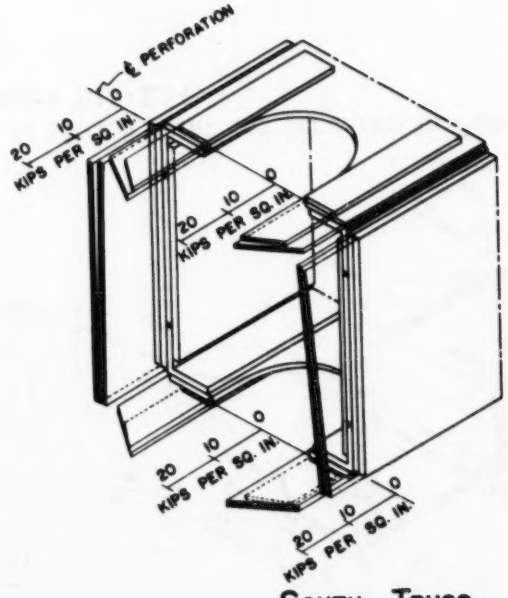
Fig. 25 -- Cantilever Load Stresses at Section 13 in Members L13-L14.



NORTH TRUSS

DATA: SECTION ON ℄ PERFORATION

GROSS AREA	167.00	SQ. IN.
COMPUTED STRESS P/A	11.80	KIPS PER SQ. IN.
AVERAGE MEASURED UNIT STRESS	10.34	KIPS PER SQ. IN.
MEASURED UNIT BENDING STRESS		
IN VERTICAL PLANE	1.32	KIPS PER SQ. IN.
IN HORIZONTAL PLANE	2.98	KIPS PER SQ. IN.



SOUTH TRUSS

DATA: SECTION ON ℄ PERFORATION

GROSS AREA	167.00	SQ. IN.
COMPUTED STRESS P/A	11.80	KIPS PER SQ. IN.
AVERAGE MEASURED UNIT STRESS	11.16	KIPS PER SQ. IN.
MEASURED UNIT BENDING STRESS		
IN VERTICAL PLANE	0.92	KIPS PER SQ. IN.
IN HORIZONTAL PLANE	1.76	KIPS PER SQ. IN.

Fig. 26 -- Full Dead Load Stresses at Section 13 in Members L13-L14.

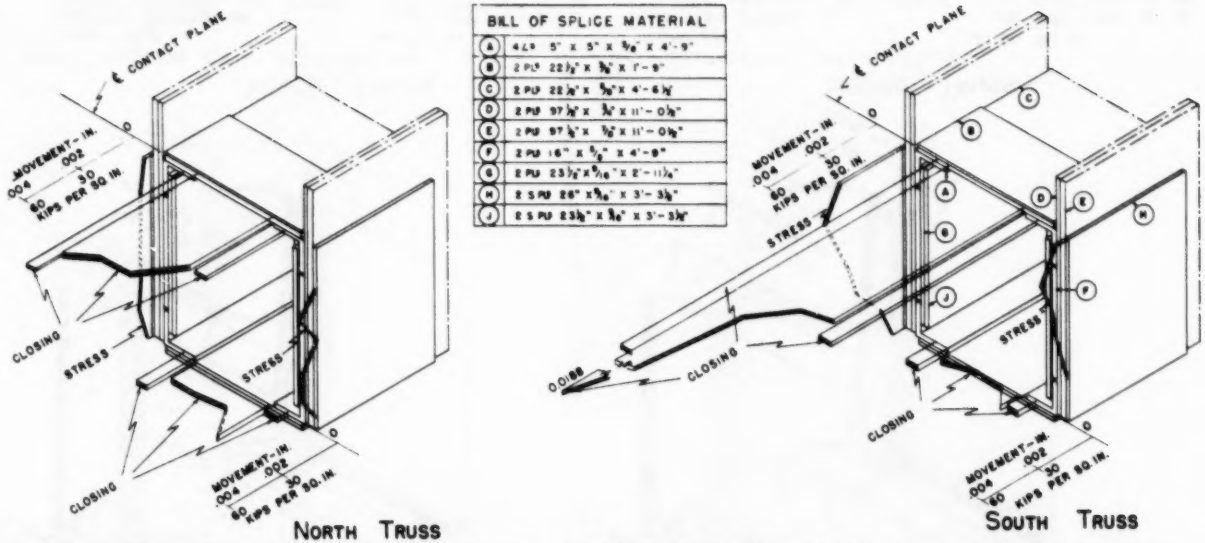

BILL OF SPLICE MATERIAL
A 4 L 5" x 5" x 5/8" x 4'-9"
B 2 PL 9 22 1/2" x 3/8" x 1'-9"
C 2 PL 9 22 1/2" x 5/8" x 4'-6 1/4"
D 2 PL 9 97 1/2" x 3/4" x 11'-0 1/8"
E 2 PL 9 97 1/2" x 1/2" x 11'-0 1/8"
F 2 PL 16" x 5/8" x 4'-8"
G 2 PL 23 1/2" x 5/16" x 2'-11 1/4"
H 2 S PL 28" x 5/8" x 3'-3 1/2"
J 2 S PL 23 1/2" x 3/4" x 3'-3 1/2"
℄ CONTACT PLANE
MOVEMENT-IN.
.004 .002
60 30
KIPS PER SQ. IN.
CLOSING
STRESS
NORTH TRUSS
SOUTH TRUSS

Fig. 27 -- Cantilever Load, Closing of Milled Joint and Stresses in Plate, Shop Splice--Section 14S Members L13-L14.

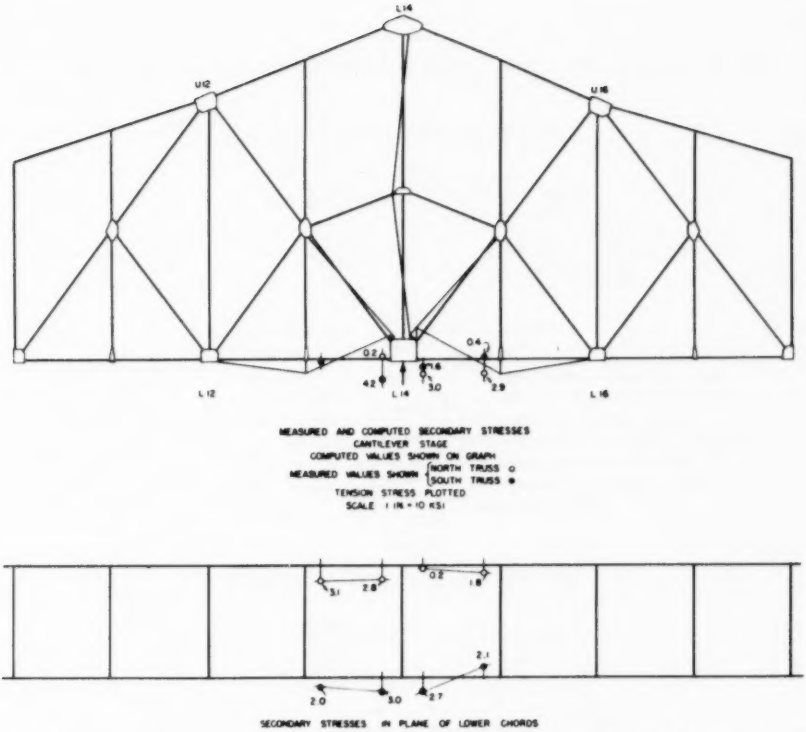


Fig. 28 -- Comparison Between Measured and Computed Bending Stresses at Cantilever Stage of Construction.

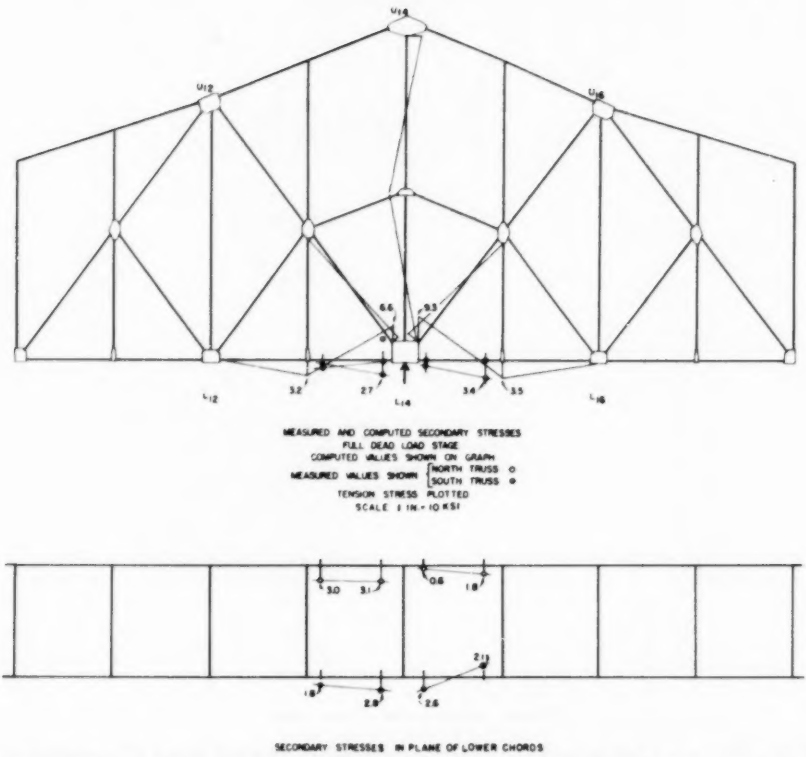


Fig. 29 -- Comparison Between Measured and Computed Bending Stresses at Final Full Load Stage.

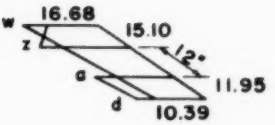
COMPUTATIONS OF AVERAGE MEASURED UNIT STRESS MEASURED UNIT BENDING STRESSES AND COVER PLATE STRESS RATIOS

SECTION 14 L13–L14 SOUTH TRUSS FULL DEAD LOAD STAGE

COVER PLATE STRESS RATIOS

$r = \frac{19.90}{15.10} = 1.32$

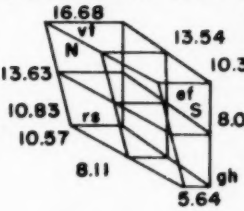
$r = \frac{19.60}{11.95} = 1.63$



STRESSES ON CENTRAL PLANES

Horizontal Vertical

13.63 13.54 2.80 2.71
10.83 10.83 2.81 2.72
8.02 8.11

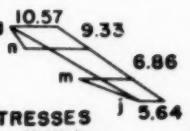


NOTE:
All stresses in Kips per sq.in.
r Denotes the ratio of stress at inside of cover plate perforation to figure obtained by interpolating between the measured stresses in the adjacent webs.

COVER PLATE STRESS RATIOS

$r = \frac{14.80}{9.33} = 1.59$

$r = \frac{13.85}{6.86} = 2.01$



MEASURED STRESSES

Cover Plates		Bars		N. Webs		S. Webs	
a	19.60	b	17.80	v	17.05	e	10.40
d	14.00	c	12.50	t	16.30	f	10.38
z	19.90	x	14.55	av.	16.68	av.	10.39
w	12.20	y	20.20	wi	13.50	wh	8.49
				ws	14.20	wo	7.26
m	13.85	k	7.42				
j	6.15	l	11.00				
n	14.80	o	12.38	r	9.86	q	5.56
q	9.43	p	9.20	s	11.28	n	5.71
				av.	10.57	av.	5.64

Fig. 30

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